CHAPTER 2: LITERATURE REVIEW

2.1 Horizontal Shear

In bridge design, it is important to develop composite action between the bridge deck and bridge girder. The horizontal shear forces that develop at the interface of the deck and girder need to be adequately transferred across the interface for composite action to be achieved. It is important to be able to predict the horizontal shear forces that will be developed at the interface and to understand the horizontal shear strength of the materials that are used.

2.1.1 Horizontal Shear Stress

The horizontal shearing stress at any point in a beam's cross section can be determined by multiple available equations. The following equation is one of the most fundamental equations for calculating horizontal shearing stress and comes from the elastic beam theory.

$$v_h = \frac{VQ}{Ib_v} \tag{2.1}$$

where:

v_h = horizontal shear stress

V = vertical shear force at section

Q = first moment of area of portion above interface with respect to neutral axis of section

I = moment of inertia of entire cross section

b_v = width of interface

This equation assumes the section is uncracked and linear elastic. Loov and Patnaik (1994) suggest equation 2.1 can be used for cracked sections if the first moment of area and moment of inertia are based on the cracked section properties.

The ACI Code 318-08, Sec. 17.5.3 specifies that in a composite member, the horizontal shear strength at contact surfaces of interconnected elements must meet the limit state:

$$V_u \leq \phi V_{nh} \tag{2.2}$$

where:

V_u = factored shear force (required strength)

V_{nh} = nominal horizontal shear resistance

ϕ = strength reduction factor

Figure 2.1 shows a free body diagram of a composite bridge deck and girder illustrating the basis of calculating the horizontal shearing stress.

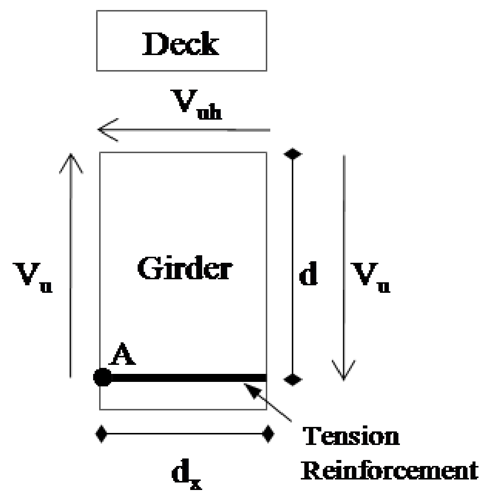

Figure 2.1 Free Body Diagram

where b_v, and V_u are previously defined and:

d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section

V_{uh}= factored horizontal shear force = $v_{uh} b_v d_x$

v_{uh}= horizontal shear stress

d_x = length of section being analyzed

The sum of forces at point A in Figure 2.1 provides:

$$V_u d_x = v_{uh} b_v d_x d \tag{2.3}$$

Rearranging Equation 2.3 then gives that the horizontal shearing stress can be calculated as:

$$v_{uh} = \frac{V_u}{b_v d} \tag{2.4}$$

Another method of calculating shear stress is presented as an alternative method in ACI 318-08, Sec. 17.5.4. It states that the horizontal shear can be determined by calculating the change in compressive or tensile force in the slab at any segment along the length of the composite section. This change in force, through satisfying equilibrium conditions, is then transferred by horizontal shear to the girder. Figure 2.2 illustrates equilibrium forces used for the basis of this method. The horizontal shear force to transfer across the interface can be defined as:

$$V_{uh} = C \tag{2.5}$$

where:

C = the change in compression force in deck

The equation for horizontal shear stress can then be defined as:

$$v_{uh} = \frac{C}{b_v l_v} \tag{2.6}$$

where:

b_v = width of interface

l_v = length of interface

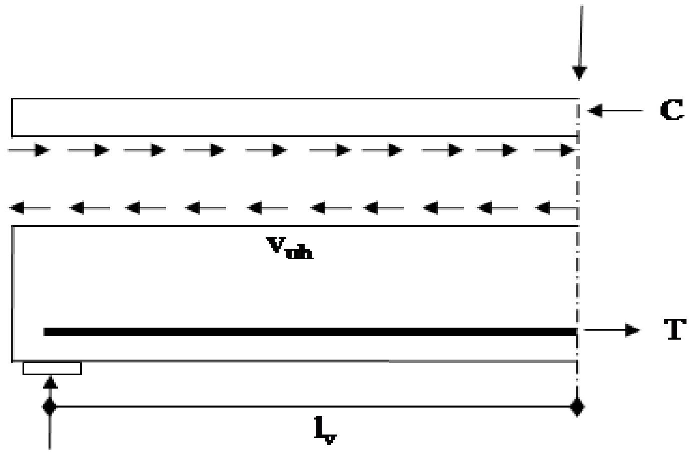

Figure 2.2 Equilibrium Forces

These three equations for calculating shearing stress seem very different. It is important to understand how the equations are related and also when each equation is appropriate to be used. A rate of change of the shear force in the deck is considered in each equation. Equation 2.6 refers to the average rate of change of force over a section with the length, l_v, where the force changes from C to zero. The term C/l_v shows this rate of change. Equation 2.1 refers to the rate of change of force in the flange at any section with the term VQ/I. By knowing that shear force can be defined as the rate of change of moment, $V_u = {dM}/{dx}$, equation 2.4 can be related. The term V_u/d is a simplification of equation 2.1 by assuming the compression zone is in the flange and the stress block depth variation is negligible. The compression force can then be described as $M/(d - \frac{a}{2})$, which makes the rate of change of the compression force $V/(d - \frac{a}{2})$.

2.1.2 Horizontal Shear Strength Equations

Many equations have been developed to determine the shear strength at the interface of a composite section. The following section presents equations from various researchers to predict interface shear strength. Throughout this section, ρ_v is defined as:

$$\rho_v = \frac{A_{vf}}{A_{cv}} \tag{2.7}$$

where:

A_{vf} = area of shear reinforcement crossing the interface

A_{cv} = area of concrete at the interface

The nominal horizontal shear strength is referred to as v_n and the term $\rho_v f_y$ is referred to as the clamping stress:

$$\rho_v f_y = \frac{A_{vf} f_y}{A_{cv}} \tag{2.8}$$

where:

f_y = yield strength of the shear reinforcement

2.1.2.1 Hanson

Research was performed by Hanson (1960) on push-off specimens and girder specimens to investigate composite action in concrete design. Different contact surface treatments were tested while concrete compressive strength was not considered. Hanson observed during his testing that the peak shear resistance was obtained when a slip of 0.005 in. occurred. The results determined the maximum shear stress resistance of a concrete precast girder and cast-in-place deck to be 500 psi for a roughened surface and 300 psi for a smooth surface. Hanson also proposed that in addition to these maximum values, 175 psi of shear capacity could be added for each percent of area of reinforcement to area of concrete crossing the interface. This addition of shear capacity was derived from push-off test results for specimens containing No.4 stirrups when 0.005 in. of slip was exhibited. Hanson stated that the use of bigger stirrups for shear reinforcement may increase the shear capacity addition.

2.1.2.2 Saemann and Washa

Tests were performed by Saemann and Washa (1964) on 42 full size T-beams to determine the strength of the shear joint between precast beams and cast-in-place slabs. The tests included 36 total different combinations varying the degree of roughness at the joint, percent of steel crossing the joint, length of shear span, the position of the joint in relationship to the neutral axis, effects of shear keys, and concrete compressive strength. The resulting equation for ultimate shear strength was as follows:

$$Y = \frac{2700}{X+5} + 300P(\frac{33-X}{X^2+6X+5}) \quad (2.9)$$

where:

Y = Ultimate Shear Stress

X = effective depth, distance of centroid of tension reinforcement from compression face of concrete

P = percent steel crossing the interface, $\frac{A_{vf}}{A_{cv}}$

The first term in this equation accounts for the equation of the curve fitting the data for specimens without steel crossing the interface. The second term then describes the effect of the addition of steel across the interface.

Effective depth and percent of steel crossing the interface were the only significant variables affecting this equation. Results showed that as the ratio of shear span to effective depth increased, the ultimate shear strength decreased. Also, as the percent of steel across the interface increased, the ultimate shear strength increased. As more steel was placed across the interface, the effect of the roughness of the joint became less, and the effect was variable throughout the tests. Because of this, the roughness was not considered in the development of equation 2.9.

2.1.2.3 Birkeland

Birkeland and Birkeland (1966) proposed the shear friction theory in a discussion of connections in precast concrete. Shear failure was described as slip along a crack in the concrete. The slip was resisted by friction, μP, caused by the external clamping force, P. If reinforcement was provided across the interface, this sliding motion developed tension, T, in the reinforcement and provided a clamping force across the interface. The roughness of the crack was described as a frictionless series of sawtooth ramps with a slope of $\tan(\theta)$. Figure 2.3 illustrates the basis of this method. An equivalent of the frictional force, μP, was then described as $T tan(\theta)$. The ultimate shear force across the interface was assumed to be reached at yielding of the reinforcement and was written as follows:

$$V_u = A_s f_y tan(\theta) \quad (2.10)$$

where:

A_s = total cross-sectional area of reinforcing across the interface

f_y = yield strength of reinforcing (≤ 60 ksi)

$tan\,\theta$ = 1.7 for monolithic concrete

= 1.4 for artificially roughened construction joints

= 0.8 to 1.0 for ordinary construction joints and for concrete to steel interfaces

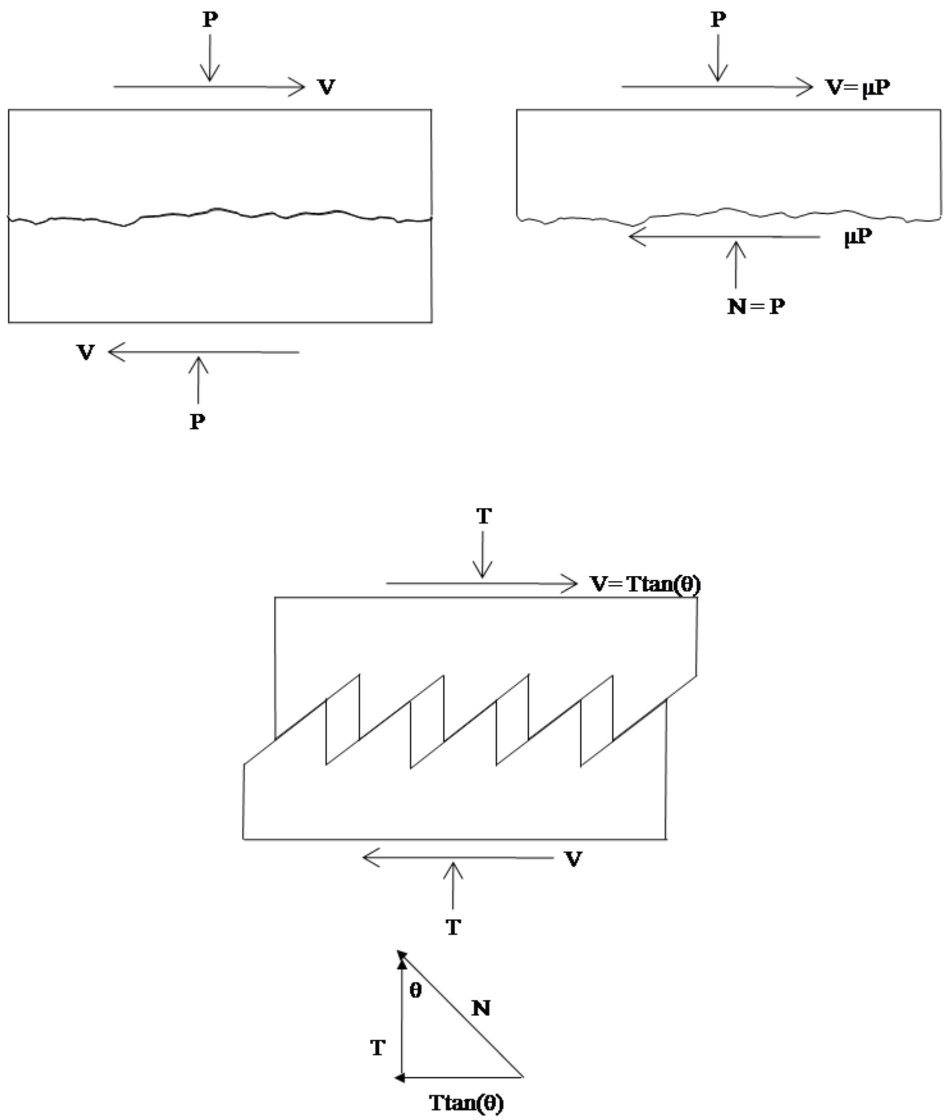

Figure 2.3 Shear friction hypothesis

This equation was limited to a maximum reinforcement size of No.6 rebar or ½ in. diameter headed studs. It was also noted that the shear was developed by friction, not by bond and that the cohesive strength was neglected to form a lower bond for test data.

2.1.2.4 Mast

Mast (1968) further discussed the shear friction theory. The equation was the same as that introduced by Birkeland but included a maximum shear stress of:

$$v_n = \rho_v f_y \tan\theta \leq 0.15 f'_c \tan\theta \tag{2.11}$$

and the friction coefficient was examined more carefully by test results and determined to be as follows:

$\tan\theta =$ 1.4 to 1.7 for a crack in monolithic concrete

= 1.4 for a rough interface between precast and cast-in-place concrete

= 1.0 for concrete cast against steel

= 0.7 for concrete against smooth concrete

ACI’s shear-friction design procedure was based on the idea of shear-friction. It became the most simple design model yet did not include the concrete’s strength effects or effects of cohesion between the two components at the interface. Loov and Patniak (1994) concluded that this model was very conservative for low clamping stresses and unsafe when considering high clamping stresses.

2.1.2.5 Shaikh

Shaikh (1978) compared previously derived equations to test data and suggested an equation for shear strength in his revisions to shear-friction provisions in PCI. The area of reinforcement normal to a plane with ultimate shear can be expressed as:

$$A_v = \frac{V_u}{\phi f_y \mu_e} \tag{2.12}$$

where:

$\phi = 0.85$ for shear

μ_e = the effective coefficient of friction

The effective friction coefficient should approach the friction coefficient (μ) as the interface capacity is reached. PCI stated the equation for effective coefficient of friction to be:

$$\mu_e = \frac{1000\lambda A_v \mu}{V_u} \tag{2.13}$$

where 1.0λ is substituted for μ, and λ was a constant used to account for the effect of concrete density:

$\lambda = 1.0$ for normal weight concrete

$\lambda = 0.85$ for sand-lightweight concrete

$\lambda = 0.75$ for all-lightweight concrete

The PCI Design Handbook (1992) based design requirements on the simplified form of these combined equations shown as follows:

$$v_u = \lambda\sqrt{1000\phi\rho_v f_y} \leq 0.25f'_c\lambda^2 \text{ and } \leq 1000\lambda^2 \text{ (psi)}$$

The result was a parabolic equation that represented test data more accurately than the linear shear-friction model.

2.1.2.6 Loov

The first to incorporate the influence of concrete strength was Loov (1978). He proposed a shear stress equation in the form:

$$\frac{v_n}{f'_c} = k\sqrt{\frac{\sigma}{f'_c}} \tag{2.14}$$

where k was a constant, and σ represented the normal compressive strength across the plane of the crack. The normal compressive strength was equal to the tensile stress from reinforcement across the crack by shear friction. The equation can then be written as:

$$v_n = k\sqrt{\rho_v f_y f'_c} \tag{2.15}$$

Most test specimens had concrete strengths of 3000psi to 4500psi, and a value of $k = 0.5$ was suggested for shear interfaces that were initially uncracked.

2.1.2.7 Walraven

Walraven (1987) performed push-off tests on a total of 88 specimens. Walraven paid close attention to the contribution of concrete's compressive strength during analysis because of a high expectation from theoretical calculations of compressive strength to play a significant role in the shear friction capacity. Only the cube compressive strength, f'_{cc} , was available for most of the test specimens so this value was the basis of data evaluation. A statistical analysis of the results from these tests determined the equation for shear friction capacity to be:

$$v_u = C_3(0.007\rho_v f_y)^{C_4} \text{ (psi)} \tag{2.16}$$

where:

$C_3 = 15.686 f'_{cc}{}^{0.406}$

$C_4 = 0.0353 f'_{cc}{}^{0.30}$

$f'_{cc} = concrete\, compressive\, strength\, of\; 5.9\; in.\, cubes$

$f'_c = 0.85 f'_{cc}$

The precision of this equation was found to be very high with an average test/calculated ratio of 1.00 for 88 test specimens and a coefficient of variation of 0.11, but because of complexity of the formulas, it is not desired to be used directly in design. Walraven developed a design chart taking into consideration the compressive strength of concrete in which the required amount of shear reinforcement could be read directly.

2.1.2.8 Mattock

Mattock has proposed several equations for horizontal shear strength throughout the years. In an experimental program performed in 1969, the objectives were to study the influence of the size and arrangement of shear reinforcement, the concrete's compressive strength, dowel action of reinforcing bars, a pre-existing crack at the interface, and the application of the shear friction theory for interface strength. Push-off test specimens were used for this study. It was determined that for low values of $\rho_v f_y$ PCI's limit of $v_n \leq 0.3 f'_c$ was very conservative. Mattock proposed a modified shear friction method including a cohesion term as follows:

$$v_n = 0.8\rho_v f_y + 400 \text{ (psi)} \tag{2.17}$$

where:

$v_n \leq 0.3f'_c$

$\rho_v f_y \geq 200$ psi

In further research, Mattock studied lightweight concrete and the application of the ACI 318-71 provisions for shear transfer strength. ACI 318-71 Section 11.15 stated shear transfer strength to be given by:

$$v_u = \phi A_{vf} f_y \mu \tag{2.18}$$

(Note that in making comparisons, ϕ was taken as 1.0 since the material strengths and specimen dimensions were accurately known.)

where:

$v_u \leq 0.2f'_c \; and$ 800 psi

and the coefficient of friction was given as:

$\mu = 1.4$ for concrete cast monolithically

$= 1.0$ for concrete placed against a hardened concrete

$= 0.7$ for concrete placed against as-rolled structural steel

Mattock concluded these provisions may be used for shear transfer strength design calculations for lightweight concrete provided that the coefficients of friction are multiplied by the following factors whereas before applying these factors, predicted values were unconservative for test data.

(1) For all-lightweight concrete having a unit weight not less than 92 lb per cubic ft, μ is multiplied by 0.75.
(2) For sand lightweight concrete having a unit weight not less than 105 lb per cub ft, μ is multiplied by 0.85.

Mattock also concluded equation 2.15 to be non-conservative for sand lightweight and all-lightweight concrete. It was proposed for shear strength of sand lightweight concrete:

$$v_u = 0.8\rho f_y + 250 \text{ psi} \tag{2.19}$$

where:

$v_u \leq 0.2f'_c \; and \; 1000$ psi

$\rho f_y \geq 200$ psi

For all-lightweight concrete:

$$v_u = 0.8\rho f_y + 200 \text{ psi} \qquad (2.20)$$

where:

$v_u \leq 0.2f'_c \; and \; 800$ psi

$\rho f_y \geq 200$ psi

2.1.2.9 Mau and Hsu

In their discussion of Walraven's shear friction capacity equation, equation 2.16, Mau and Hsu (1988) proposed a similar equation to that proposed by Loov, equation 2.15, where $k = 0.66$ for both initially cracked and initially uncracked specimens. They also proposed a limit on shear stress of $v_n/f'_c \leq 0.33$. The cube compressive strength, f'_{cc}, was converted to concrete compressive strength by multiplying by a factor of 0.85 as proposed by Walraven. It was stated this equation was advantageous in that the equation was dimensionless and deviates the least from Mattock's modified shear friction equation.

2.1.2.10 Loov and Patnaik

Loov and Patnaik (1994) derived one equation for horizontal shear strength of composite concrete beams with a rough interface to replace four of the equations required by the ACI Code at the time. The derived equation combined the effect of clamping stress and concrete strength. Horizontal shear strength of composite beams without stirrups was analytically approximated as:

$$v_{no} = 0.6\sqrt{15f'_c} \text{ (psi)} \qquad (2.21)$$

A continuous curve fitting equation was then derived by combining this equation with Loov's previous findings in equation 2.15. Horizontal shear strength was then calculated as:

$$v_n = k\lambda\sqrt{(15 + \rho_v f_y)f'_c} \leq 0.25f'_c \text{ (psi)} \quad (2.22)$$

where $k = 0.6$

This provided a good lower bound for concrete strengths ranging from 2500 to 7000 psi. It was suggested for routine design in composite construction to use $k = 0.5$ to provide a lower shear strength accounting for a possibly smoother interface. The factor λ varied as listed in 2.1.2.5 for lightweight concrete designs.

2.1.2.11 Hwang, Yu and Lee

The study done by Hwang, Yu and Lee (2000) compared formulas required by ACI 318-95 with a softened strut and tie model of 147 experimental specimens. The proposed theory was contrary to that of the shear-friction theory, predicting that interface failure is caused by crushing of the concrete in the compression struts crossing the interface.

2.1.2.12 Kahn and Mitchell

50 push-off tests were performed by Kahn and Mitchell (2002) to study if ACI's shear friction theory was applicable to high strength concrete. The tests were performed using concrete strengths of 6800 psi to 17900 psi and shear reinforcement ratios ranging from 0.37 to 1.47 percent. It was concluded that the shear friction theory provisions from ACI provided conservative estimates of high strength concrete's interface shear strength. The equation proposed by Kahn and Mitchell incorporated a frictional component, $\mu = 1.4$, and a component for bond, $0.05f'_c$ and better predicted results for normal and high strength concrete:

$$v_n = 0.05f'_c + 1.4\rho_v f_y \leq 0.2f'_c \text{ psi} \quad (2.23)$$

where the factor of 1.4 came from the friction coefficient for a rough joint. It was also recommended that in order to limit slip along smooth cracks, f_y was limited to 60 ksi. As seen in the equation, the upper limit of 20% of the concrete compressive strength was still placed on the horizontal shear strength, but it was recommended that the shear stress upper limit of 800 psi be eliminated.

2.1.2.13 Kahn and Slapkus

Kahn and Slapkus (2004) performed research on interface shear strength in composite T-beams with high strength concrete. Six T-beams were cast using 12 ksi concrete for the precast webs, and three of the cast-in-place flanges used 7 ksi, while the other three used 11 ksi concrete. The ratio of transverse reinforcement crossing the joint ranged from 0.19 to 0.37 percent. It was concluded that the current design provisions from AASHTO LRFD (2nd Edition) and ACI 318-02 could be applied to concrete strengths up to 11 ksi. It was also concluded that equation 2.22 developed by Loov and Patnaik provided the most accurate prediction of interface shear strength.

2.1.3 ACI 318-08/318R-08 for Horizontal Shear

Horizontal shear design for ACI 318-08 Building Code is based on the following fundamental equation:

$$V_u \leq \phi V_{nh}$$

where:

V_{nh} = nominal horizontal shear strength (lb)

V_u = factored shear force at the section considered (lb)

ϕ = 0.75 for shear

The value of V_u may be taken as the factored shear force at the section under consideration or by computing the change in compression or tension force at that section. The design strength of the section may then be calculated by following the series of equations shown below.

If:

$$V_u > \phi 500 b_v d \tag{2.24}$$

then:

$$V_{nh} = A_{vf} f_y \mu \leq min \begin{cases} 0.2f'_c A_c \\ 800A_c \end{cases} \tag{2.25}$$

Else if:

$$V_u \leq \phi 500 b_v d \tag{2.26}$$

then three surface preparations are considered to determine design strength.

The first is when the contact surfaces are clean, free of laitance, and intentionally roughened then the following equation may be used:

$$V_{nh} = 80 b_v d \tag{2.27}$$

The second condition is when the contact surfaces are not intentionally roughened and minimum horizontal shear ties are provided. Then the previous equation may be used where the minimum reinforcement meets the equation:

$$A_{vmin} = 0.75\sqrt{f'_c}\frac{b_w s}{f_{yt}} \geq \frac{50 b_w s}{f_{yt}} \tag{2.28}$$

The third surface preparation includes when the minimum horizontal shear ties are provided and the contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately ¼ in. The design equation is then as follows:

$$V_{nh} = (260 + 0.6\rho_v f_y)\lambda b_v d \tag{2.29}$$

For the above equations:

b_v = width of cross section at contact surface being investigated for horizontal shear (in.)

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement (in.) but not less than 0.80h for prestressed members

A_{vf} = area of shear friction reinforcement (in^2)

f_y = specified yield strength of reinforcement (psi)

μ = coefficient of friction

= 1.4λ for concrete placed monolithically

= 1.0λ for concrete placed against hardened concrete intentionally roughened to approximately ¼ in.

= 0.6λ for concrete placed against hardened concrete not intentionally roughened

= 0.7λ for concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars

λ = lightweight modification factor

= 1.0 for normal weight concrete

= 0.85 for sand-lightweight concrete

= 0.75 for all lightweight concrete

A_c = area of concrete resisting shear (in^2)

s = center to center spacing of reinforcement (in.)

f_{yt} = specified yield strength of transverse reinforcement (psi)

ρ_v = ratio of tie reinforcement area to area of contact surface

Commentary in Section 11.6.3 states that the equations are based on the shear-friction model and give a conservative prediction of the shear-transfer strength. The code allows other relationships that provide a closer estimate of shear-transfer to be used.

2.1.4 AASHTO LRFD Bridge Design Specifications (4th Edition)

Design equation used in AASHTO LRFD Bridge Design Specification was examined in this research. The design equation follows a similar approach as that used in ACI Building Code. The design for horizontal shear is based upon:

$$V_{ri} \geq V_{ui}$$

$$V_{ri} = \phi V_{ni}$$

where:

V_{ri} = factored interface shear resistance (k)

V_{ui} = factored interface shear force due to total load based upon applicable strength and extreme load combinations (k)

V_{ni} = nominal interface shear resistance (k)

ϕ = resistance factor for shear

= 0.90 for normal weight concrete

= 0.70 for lightweight concrete

The nominal shear resistance of the interface plane shall be calculated as:

$$V_{ni} = cA_{cv} + \mu\left(A_{vf} f_y + P_c\right) \le \min\begin{cases} K_1 f'_c A_{cv} \\ K_2 A_{cv} \end{cases} \quad (2.30)$$

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})

c = cohesion factor (ksi)

μ = friction factor

A_{vf} = area of shear reinforcement crossing the shear plane within the area A_{cv}

f_y = yield stress of reinforcement but not to exceed 60 (ksi)

P_c = permanent net compressive force normal to the shear plane

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear

K_2 = limiting interface shear resistance (ksi)

Values of c, μ, K_1, and K_2 depend on surface preparation and the use of lightweight or normal weight concrete. The $K_1 f'_c A_{cv}$ equation is to prevent shearing or crushing of aggregate along the shear plane while $K_2 A_{cv}$ was derived due to lack of data provided for values above limiting K_2 values. The value of these variables for each circumstance of surface preparation and type of concrete are as follows:

For cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with the surface intentionally roughened to an amplitude of 0.25 in.:

$c = 0.28$

$\mu = 1.0$

$K_1 = 0.3$

$K_2 = 1.8$ for normal weight concrete

$= 1.3$ for lightweight concrete

For lightweight concrete placed monolithically or nonmonolithically, against a clean concrete surface, free of laitance with the surface intentionally roughened to an amplitude of 0.25 in:

$c = 0.24$

$\mu = 1.0$

$K_1 = 0.25$

$K_2 = 1.0$

For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.

$c = 0.24$

$\mu = 1.0$

$K_1 = 0.25$

$K_2 = 1.5$

For concrete placed against clean, free of laitance surface that is not intentionally roughened:

$c = 0.075$

$\mu = 0.6$

$K_1 = 0.2$

$K_2 = 0.8$

The second term in the nominal interface shear resistance equation, $\mu(A_{vf} f_y + P_c)$, stems from a pure shear friction model. The net clamping force is considered to be the tension formed in the reinforcing steel, $A_{vf} f_y$, plus the normal force, P_c, that applies compression to the interface. Multiplying the net clamping force by a friction coefficient term transforms the clamping force to a shear force at the interface. The first term of the equation, cA_{cv} , modifies the pure shear friction model to include shear strength from cohesion and aggregate interlock at the interface. This modification was made due to evidence from experimental data that cohesion and aggregate interlock do have an effect on the interface shear strength.

The three interface shear strength equations are based on experimental data from normal weight concrete with strengths ranging from 2.5 ksi to 16.5 ksi and lightweight concrete with compressive strengths ranging from 2.0 ksi to 6.0 ksi. Limiting the value of f_y of the shear reinforcing steel to 60 ksi comes from experimental data showing the use of higher f_y values only overestimated the horizontal shear strength of the interface.

2.1.4.1 Wallenfelsz

Wallenfelsz (2006) performed 29 push-off tests to analyze AASHTO's interface nominal shear resistance equation for specimens made of precast deck panels with a grouted interface. Wallenfelsz concluded equation 2.30 should be broken into two separate equations using the maximum of the two components as the nominal shear resistance at the interface:

$$V_n = max \left| \begin{matrix} cA_{cv} \\ \mu(A_{vf} + P_n) \end{matrix} \right. \tag{2.31}$$

It was concluded resistance provided from friction did not occur until the crack at the interface was formed and the cohesion bond was broken. Using the maximum of these two equations provided accurate predictions of test data especially as the quantity of shear reinforcement in the specimens increased.

2.2 Summary of Literature Review

This chapter presented the evolution of current code equations for calculating interface horizontal shear strength. Many equations have been proposed and many revisions of previous equations have been made through the years. While precast concrete girders with cast-in-place concrete decks have been examined thoroughly in past research, not much research has been done on the use of high strength lightweight concrete for these composite systems. The following research was performed to examine the application of current code equations to the use of high strength lightweight concrete.